HENDERSON

Investigation of a
Steel Highway Bridge

Civil Engineering

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1906

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INVESTIGATION

OF A

STEEL HIGHWAY BRIDGE

BY

CHARLES ELLIOTT HENDERSON

THESIS

FOR

DEGREE OF BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

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U N I V E R S I T Y   O F   I L L I N O I S

May 26, 1906

This is to certify that the thesis prepared under the immediate direction of Instructor C. W. Malcolm by

CHARLES ELLIOTT HENDERSON

entitled INVESTIGATION OF A HIGHWAY BRIDGE

is approved by me as fulfilling this part of the requirements for the Degree of Bachelor of Science in Civil Engineering.

Ira O. Baker

Head of Department of Civil Engineering

The Gilbert Street Bridge, Danville, Illinois.

Introduction.

This bridge crosses Vermillion River at Danville, Illinois, on the line of Gilbert Street, the main thoroughfare running south from the city; and connects Danville with South Danville and Tilton, and is locally known as the "Gilbert Street Bridge". It was built in 1893 by the Chicago Bridge and Iron Works for the Township of Danville, City of Danville, and Vermillion County.

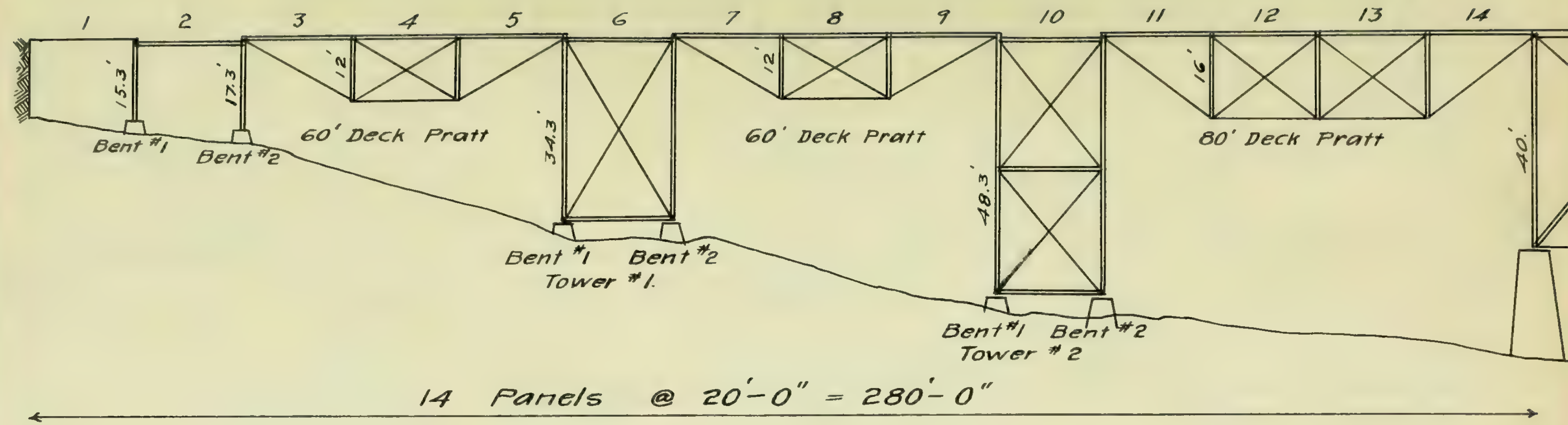
The structure is 1098 feet long, and, with the exception of the height of the towers, is symmetrical about the center. The center pier is skewed and supports the ends of similar spans, which consist of Warren deck trusses with sub-verticals. In these spans the longer trusses are 273 feet long, and are composed of

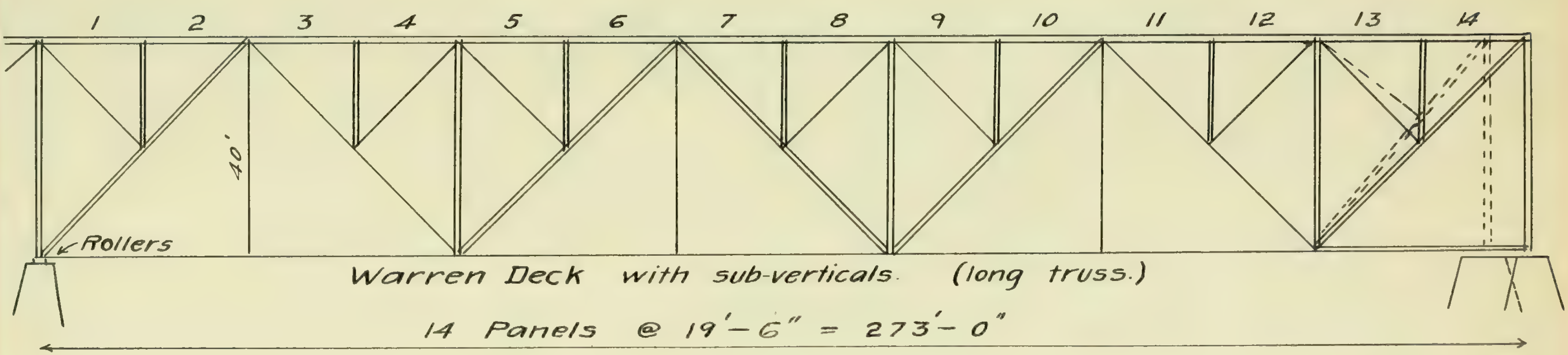
fourteen panels of 19 feet and 6 inches each. The shorter trusses are 263 feet and 8 inches long and are composed of thirteen panels of 19 feet and 6 inches each, and one panel of 10 feet and 2 inches. Besides these spans there are two 80 foot deck Pratt trusses, four 60 foot deck Pratt trusses, four towers, and four bents. The distance center to center of trusses throughout the length of the bridge is 20 feet. The wagon-way is 22 feet wide, and there is a foot way, 4 feet and 6 inches wide, on each side of the bridge. Plates No. 1 and No. 2 show a general outline of the bridge.

The bridge was built to carry heavy teaming. The heaviest teaming being that due to the coal traffic into Danville from the south, and the brick traffic from Danville south. In 1901 the Illinois Traction System obtained permission to operate cars over it, and now have double tracks the entire length of the bridge. The franchise requires that there shall

not be more than one car on the bridge at a time. When coal is hauled, the load consists of one locomotive and one or two coal cars. In addition to this there is, of course, the teaming and foot traffic.

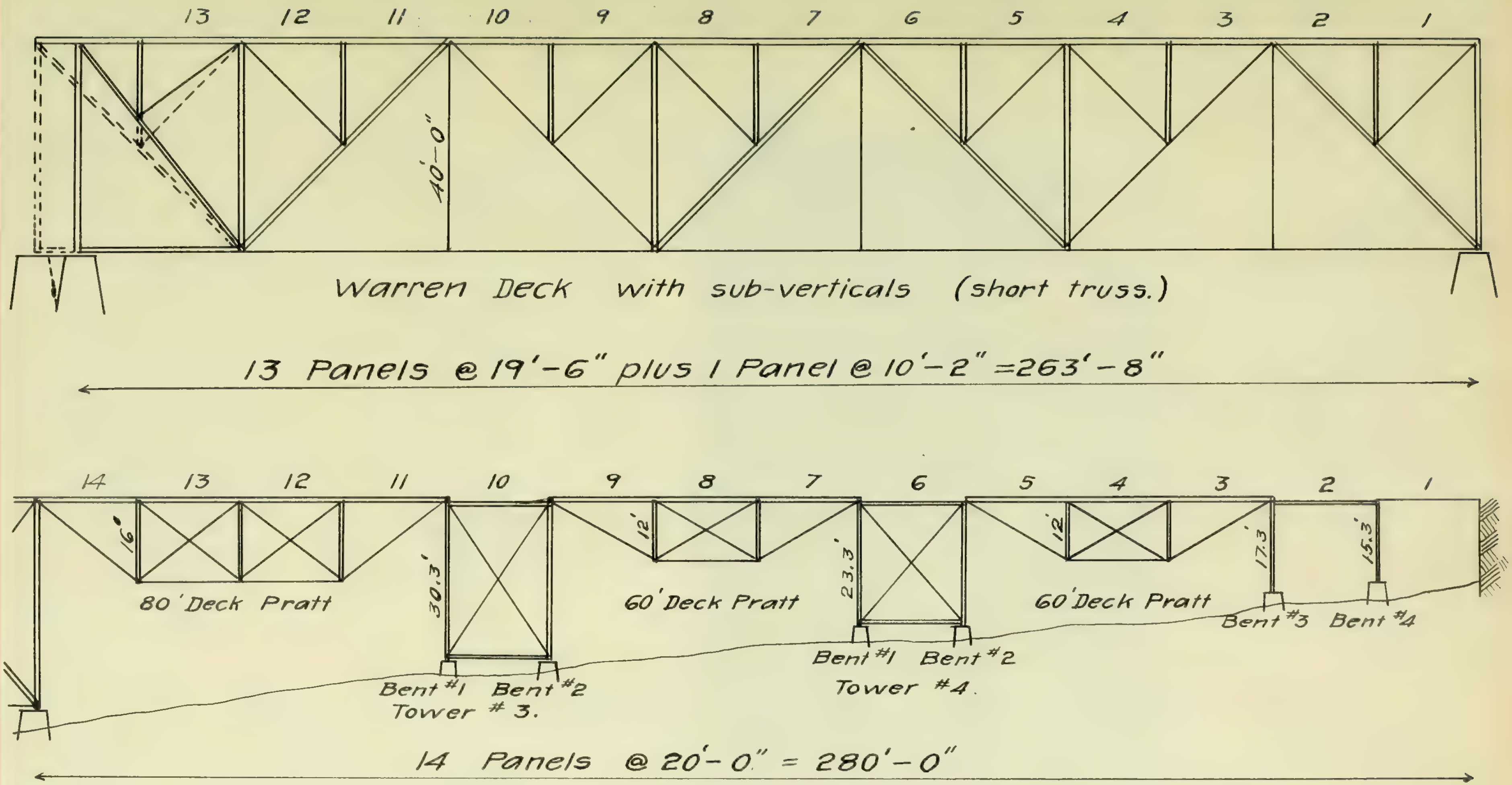
On account of the length of the bridge, this investigation, is limited to a report of the present physical condition, and to the efficiency of some of the more important members.





Elevation of North Half of Bridge.

Plate No. 1



Elevation of South Half of Bridge

Plate No. 2

View No. 1.

View of the extreme south end of the bridge, showing the two bents, tower, all of first 60-foot Pratt truss, and part of the second.

View No. 2

View near south end showing part of second 60-foot Pratt truss, the second tower, the 80-foot Pratt truss, and part of the Warren truss.

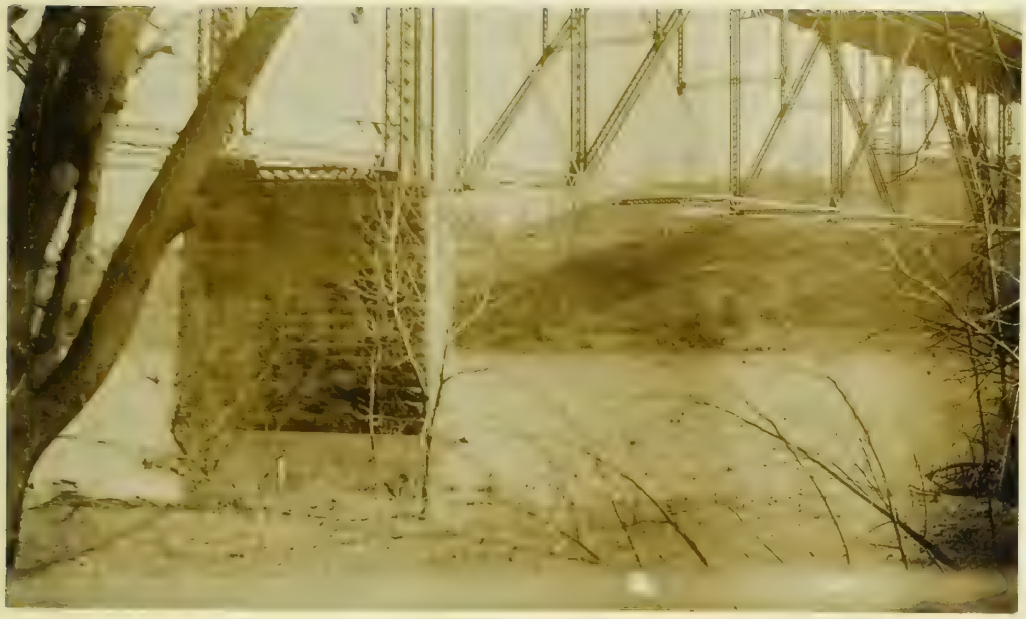
View No. 3

View of the south long span (Warren truss), and the center pier.

View No. 4.

Center pier supporting skew ends of long spans.

View No. 5



Near view of the center pier.

View No. 6

The North long span and the pier supporting the north end of the same span.

Specifications

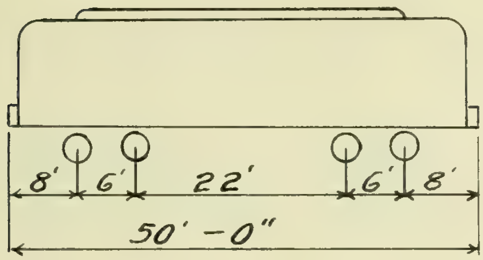
The bridge is investigated under Cooper's "General Specifications for Steel Highway and Electric Railway Bridges", 1901 Edition.

Class A1 and Class A2, termed "City Bridges", have the following requirements:— "For the floor and its supports, on any part of the roadway or on each of the street car tracks, a concentrated load of 24 tons on two axels of 10 feet centers, (assumed to occupy 12 feet in width for a single line, and 22 feet for a double line), and upon the remaining portion of the floor, including foot walks, a load of 100 pounds per square foot." The car loading in these classes is the same as in Class E1, the latter being termed, "Electric Railway Bridges with Heavy Equipment". The car loading on this bridge can be classed as Heavy Equipment, as will be seen by Plates No 3 and No 4. Therefore

the bridge is investigated under Class A1, with the following loading:- Spans up to 100 feet, - 1800 pounds per lineal foot of each track, plus 100 pounds per square foot of remaining floor space. Spans 200 feet or over, - 1200 pounds per lineal foot of each track plus 80 pounds per square foot of remaining floor space.

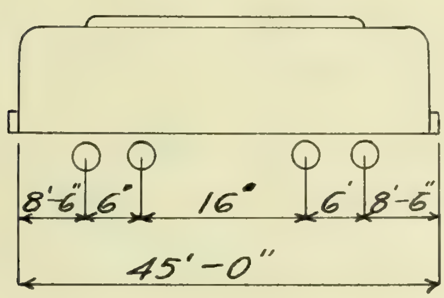
Plate No. 3

Limited Car



Weight of car	=	52 000	pounds.
Live load	=	12 000	"
Total	=	64 000	"
Wheel load.	=	8 000	"

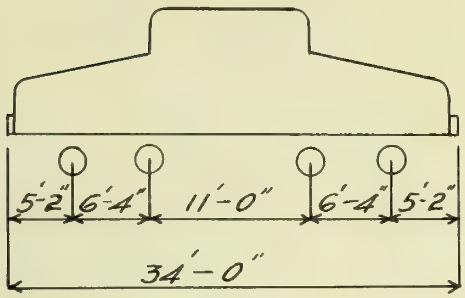
Georgetown Car



Weight of car	=	46 000	pounds.
Live load	=	10 000	"
Total	=	56 000	"
Wheel load	=	7 000	"

Plate No. 4

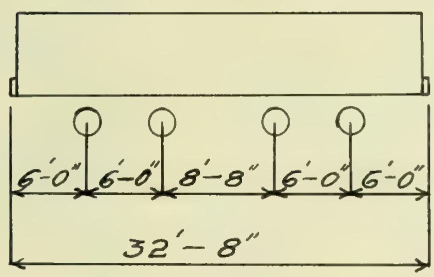
Locomotive



Total weight = 72 000 pounds

Wheel load = 9 000 "

Coal Car

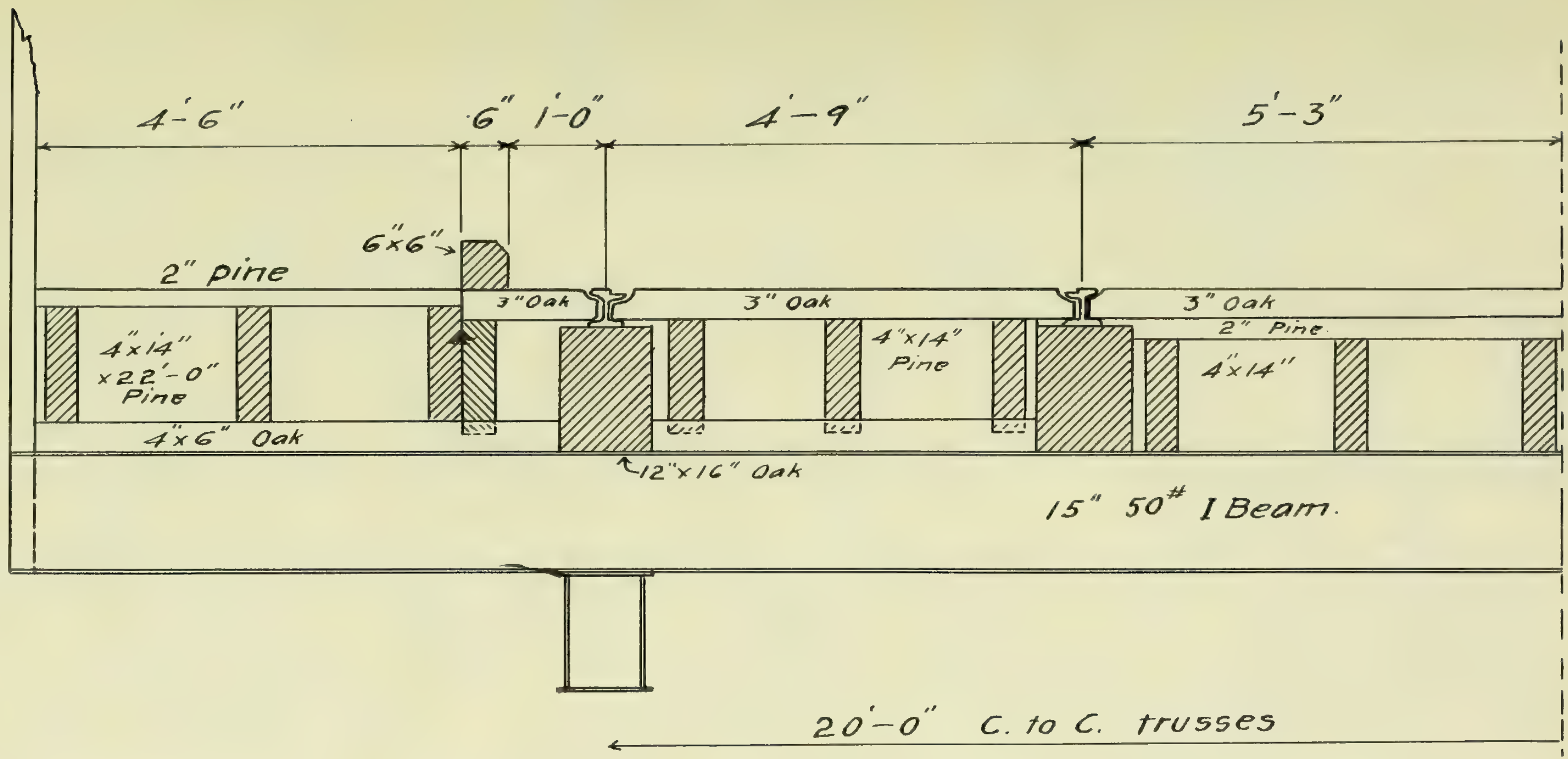


Weight of car = 23 000 pounds.

Capacity = 36 000 "

Total = 59 000 "

Wheel load = 7 400 "



Section :- One-Half Floor System

Plate No. 5

Efficiencies.

In computing the efficiencies of the different members, the first members to be considered will be the stringers and floor beams. Then beginning at the ends, the different members will be considered according to their position from the end to the center. To show the methods for obtaining the efficiencies, the work for the stringers, floor-beam, and one other member will be shown. After that, only diagrams will be used, showing composition, stresses, and efficiencies of the members.

The efficiency of the member will be either:- (1) the allowable unit stress for dead and live loads, divided by the actual unit stresses for dead and live loads; or (2) the average allowable unit stress for dead and live loads, plus 10 percent (excess for weight of member and eccentric loading), plus 25 percent (excess for wind);

divided by the actual unit stress for dead and live loads, plus (algebraically) the unit stress due to the weight of the member, plus (algebraically) the unit stress due to eccentric loading, plus (algebraically) the unit stress due to direct and flexural wind stresses. The smaller value is the efficiency. It should be noted that if the stress due to the weight and eccentric loading is less than 10 percent, or the stress due to the wind is less than 25 per cent of the allowable stress due to the dead and live loads, it is neglected (Specifications, paragraphs 52 and 55).

(1) $\frac{S_1}{S} = E$ or.

(2) $\frac{S_1 + S_1wt + S_1e + S_1w}{S + Swt + Se + Sw} = E.$

Where E = Efficiency.

S_1 = Allowable unit stress for dead and live loads.

S = Actual " " " " " " "

S_1wt = 10 per cent of S_1, for stress due to weight.

S_1e = 10 " " " S_1, " " " " eccentricity.

S_1w = 25 " " " S_1, " " " " wind.

Swt = actual unit stress due to weight of member

S_e = Actual unit stress due to eccentricity.

S_w = " " " " " wind.

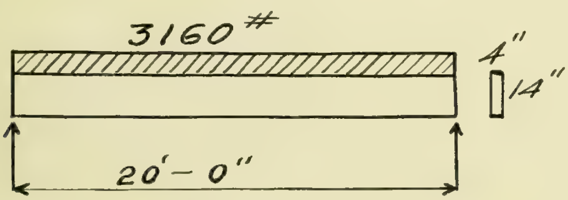
Stringer (Roadway).

Dimensions - 4"x14"x 20'-0"

Load = 100 pounds per sq. ft.

Area over one stringer = 20 x 1.58 = 31.6 sq ft.

Weight on floor = 100 x 31.6# = 3160 lbs.



$$S = \frac{Mc}{I}$$

$$M = \frac{Wl}{8} = \frac{3160 \times 20 \times 12}{8} = 94700 \text{ ins. lbs.}$$

$c = 7$ ins.

$$I = \frac{bd^3}{12} = 915$$

$$S = \frac{94700 \times 7}{915} = 725 \,\#/\square''$$

Allowable stress for wood = 800 #/□"

$$Eff. = \frac{800}{725} = 1.10$$

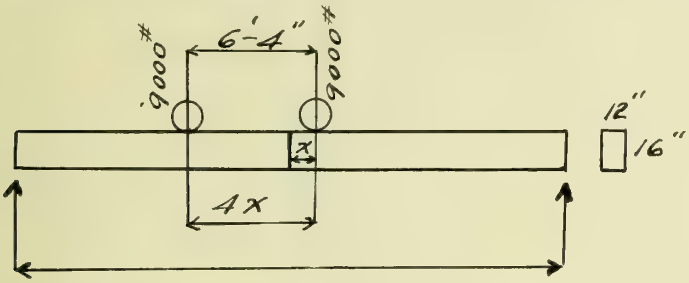
Stringer (Track)

Dimensions - 12" x 16" x 20'-0"

Load - Locomotive (see plate #4.)

Total weight = 72000 lbs.

Wheel load = 9000 lbs.



$M = 765\,000$ ins. lbs.

$c = 8$ ins.

$I = 4090$

$$S = \frac{Mc}{I} = \frac{765\,000 \times 8.0}{4090} = 1495 \text{ lbs/sq in.}$$

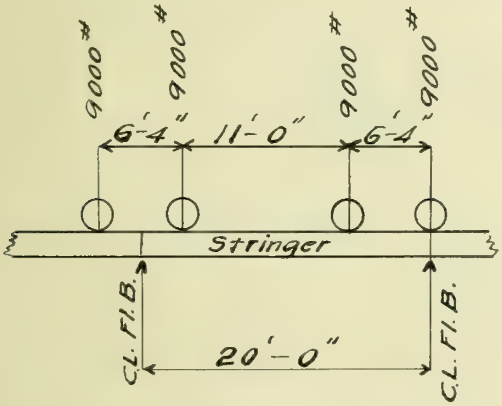
Allowable stress for wood = 800 lbs/sq in.

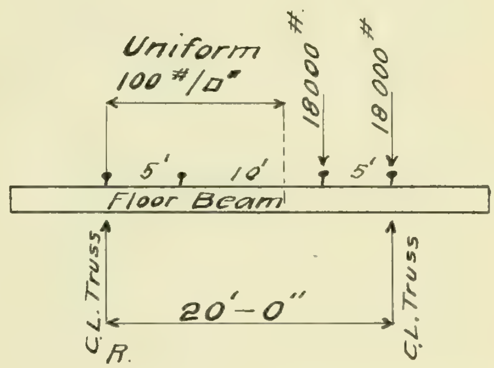
$$\text{Eff.} = \frac{800}{1495} = .54$$

Floor Beam.

15" 50# I Beams 32'-6" long.

Load = Locomotive with center of truck directly over floor beam on one track and remainder of floor between trusses with uniform load of 100 lbs per sq. ft.





$$R = \frac{18000 \times 5}{20} + \frac{11.5 \times 20 \times 100 \times 14.25}{20} = 20900 \text{ lbs.}$$

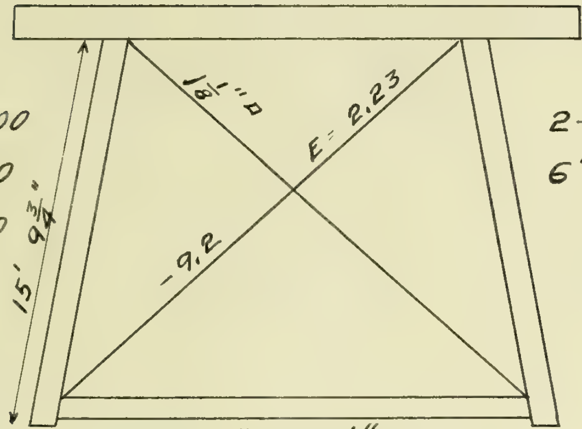
$$M = 20900 \times 10 \times 12 - 20000 \times 5 \times 12 = 1\,310\,000 \text{ ins. lbs.}$$

$$S = \frac{1\,310\,000 \times 7.5}{483.4} = 20350$$

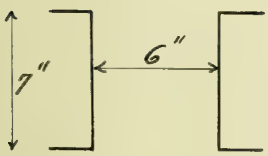
Allowable $S = 13000$

Eff. $= 13000 \div 20\,350 = .638$

Bent No. 1.



Post



2 − 7" − 9.75# [s laced.

dl + 16 800	A = 2 × 2.85 = 5.7 □"
ll + 45 800	I = 2 × 21.1 = 42.2
W + 4500	l = 15.8'
	r = 2.72"

$$dl\ P = 22\,000 - 80\frac{l}{r} = 22\,000 - 80\frac{15.8\times 12}{2.72} = 16420\ \#/\square''$$

dl area req. = 16800 ÷ 16420 = 1.02 □"

$$ll\ P = 11000 - 40\frac{l}{r} = 11000 - 40\,\frac{15.8\times 12}{2.72} = 8210\ \#/\square''$$

ll area req. = 45800 ÷ 8210 = 5.58 □"

Average allow. unit stress = 62600 ÷ 6.6 = 9500 #/□"

Actual unit stress. = 62 600 ÷ 5.7 = 11000 #/□"

Direct wind stress = + 4500

Unit stress = 4500 ÷ 5.7 = 790 #/□"

This is neglected as it is less than 25 percent of the allowable stress due to dead

and live loads. The efficiency then equals, average allowable unit stress divided by actual unit stress $= 9500 \div 11000 = .86$.

There is no eccentricity in the member, and as the post is almost vertical, the stress due to its weight is not considered. When necessary the eccentricity and weight will be considered as follows:-

Stress due to eccentric loading is

$$f_e = \frac{My_1}{I - \frac{Pl_2}{10E}} = \frac{Pey_1}{I - \frac{Pl_2}{10E}}$$

In which

P = force due to load in pounds.

e = amount of eccentricity in inches.

y_1 = distance from neutral axis to fibre considered.

l_1 = length of member in inches.

Stress due to the weight of member is

$$f_w = \frac{My_1}{I - \frac{Pl_2}{10E}} = \frac{\frac{1}{8} Wl_1 \sin\theta\, y_1}{I - \frac{Pl_2}{10E}}$$

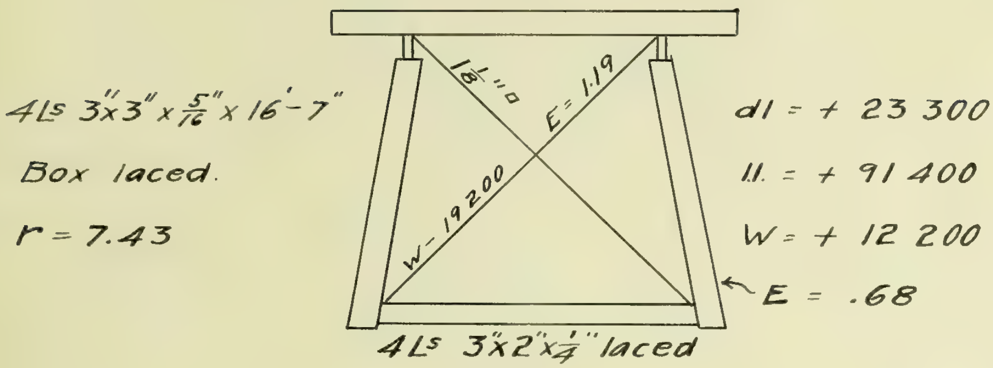
In which

W = total weight of member in pounds.

θ = angle with the vertical.

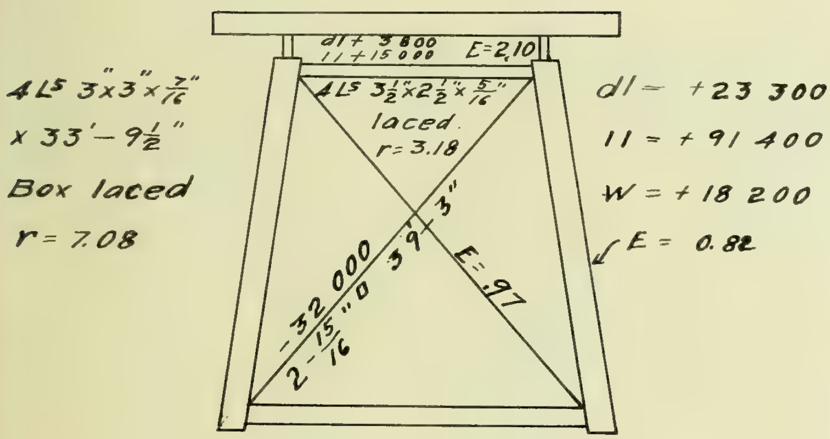
Plate No. 6

Bent No. 2



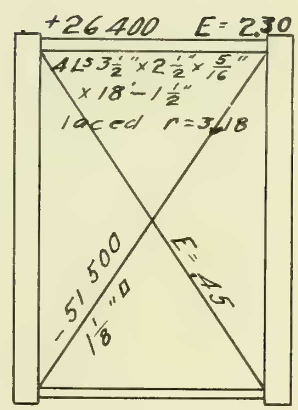
Tower No. 1

Bents #1 and #2



Lateral

Bracing

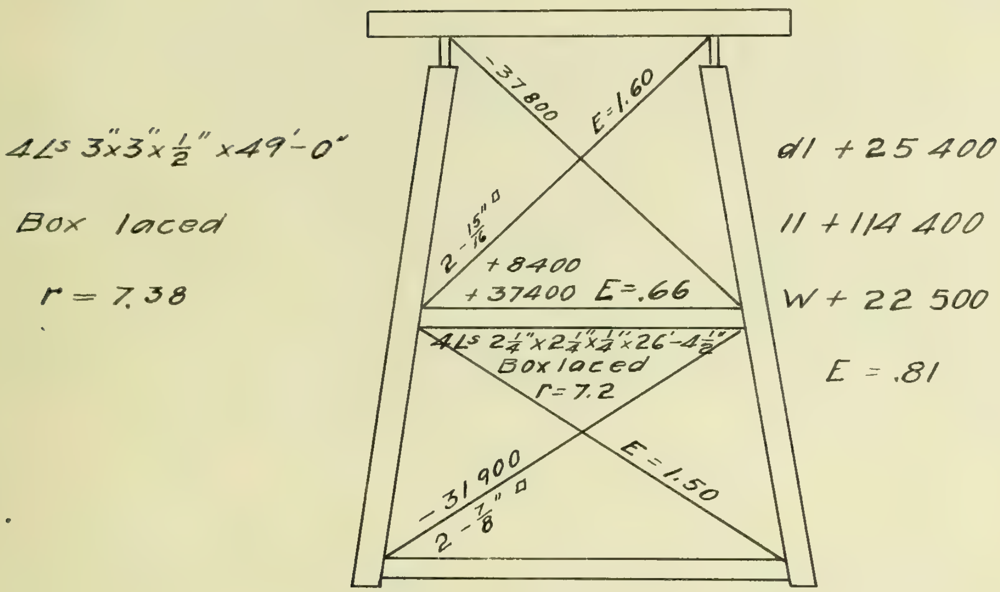


Longitudinal

Bracing

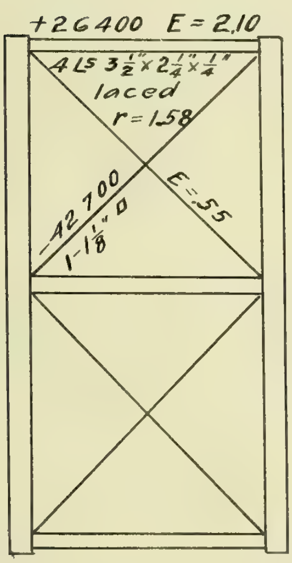
Plate No. 7

Tower No. 2



Lateral

Bracing

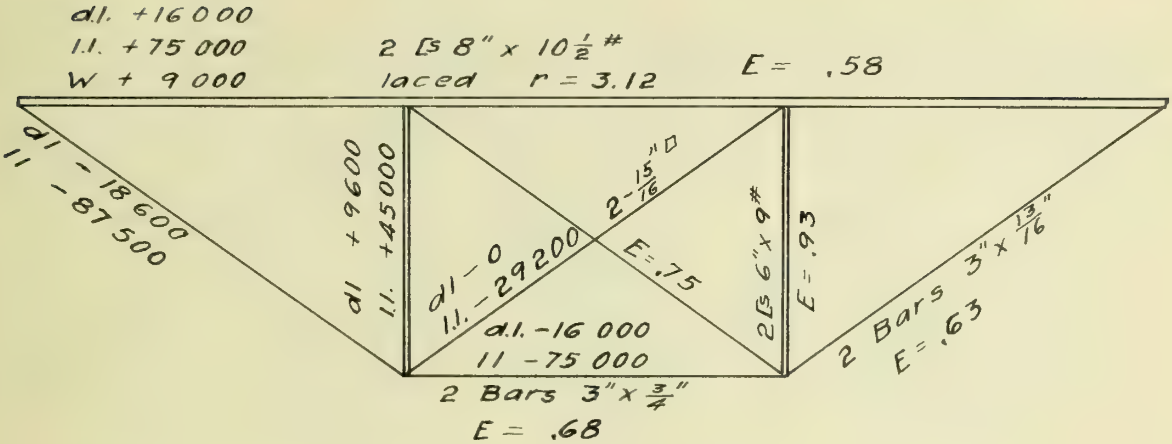


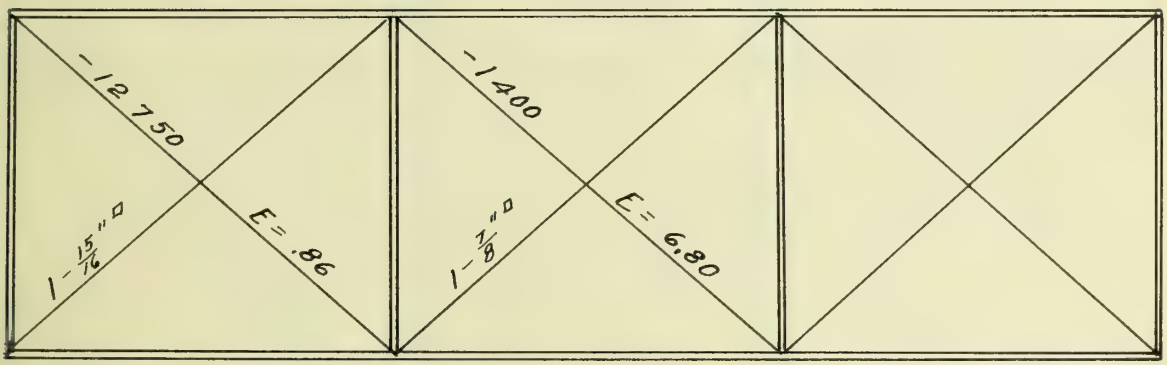
Longitudinal

Bracing

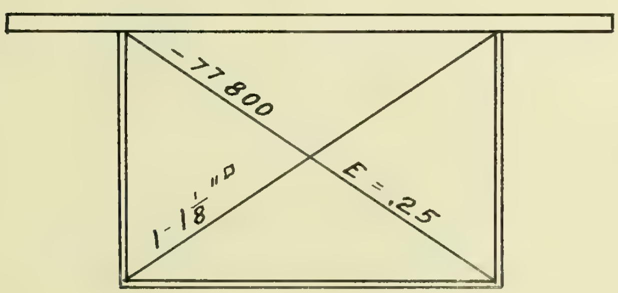
Plate No. 8

60 Foot Deck Pratt.





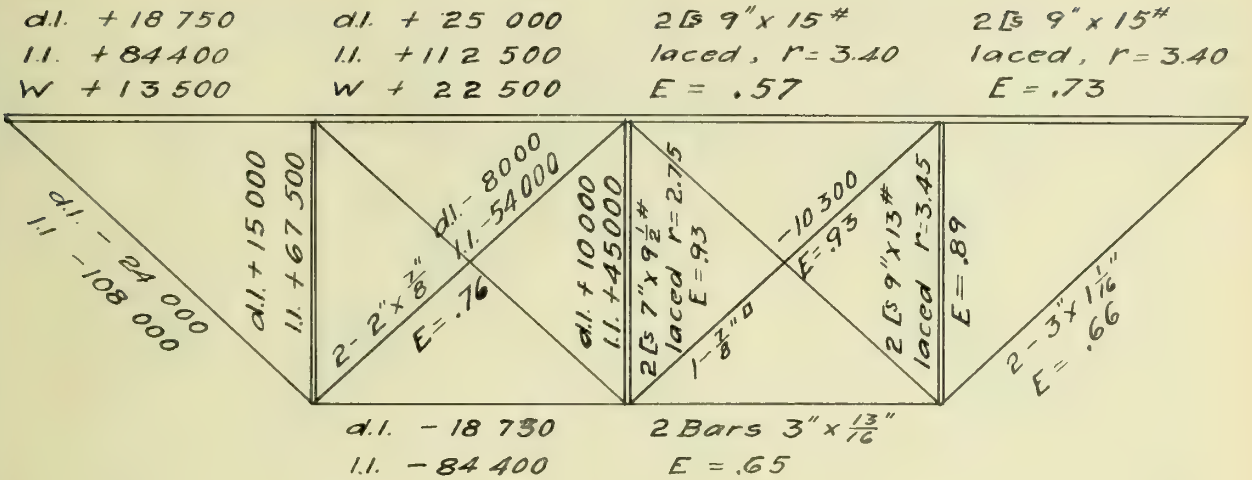
Top Laterals.

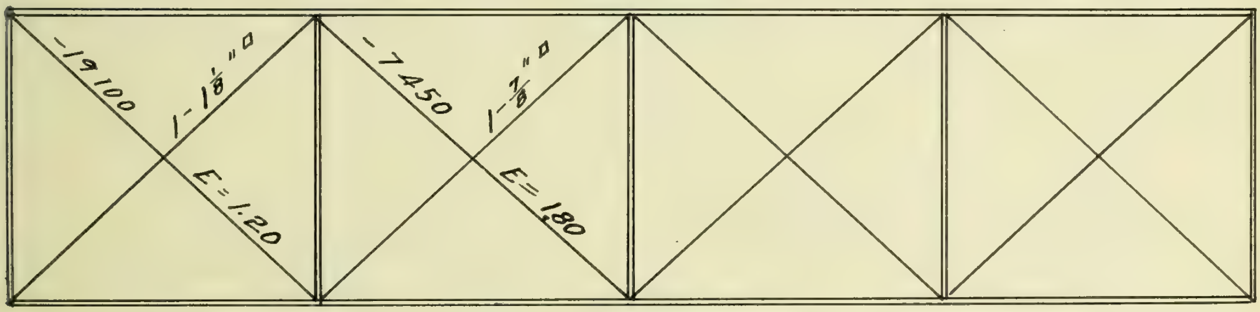


Sway Bracing

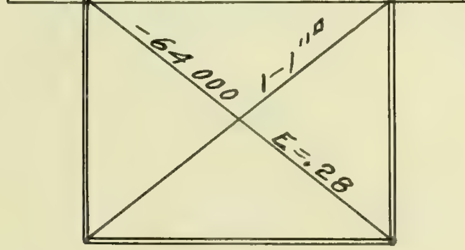
Plate No. 9

80 Foot Deck Pratt





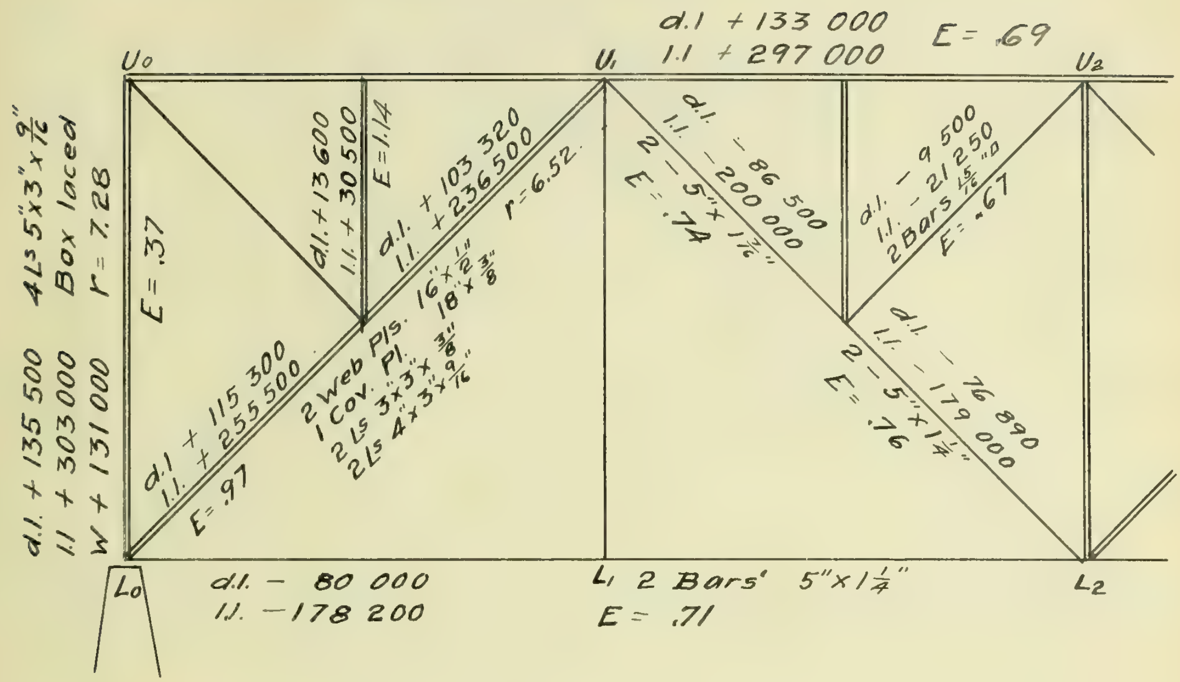
Top Laterals.



Sway Bracing.

Plate No. 10

Warren Deck



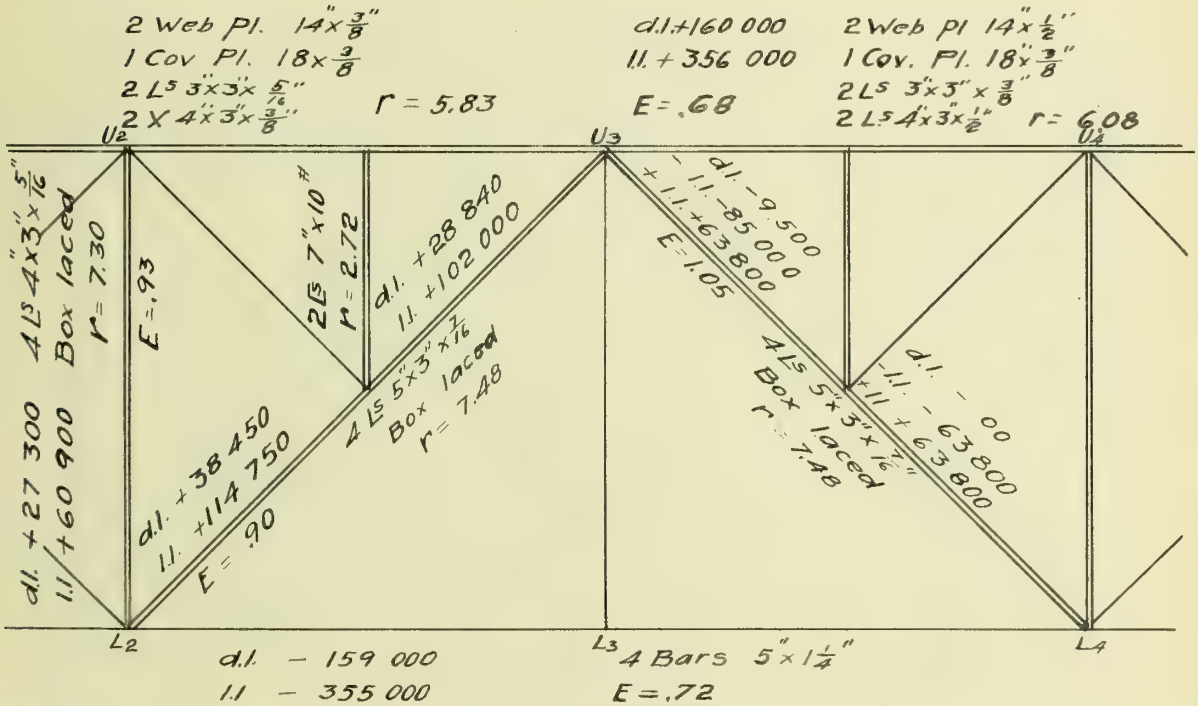
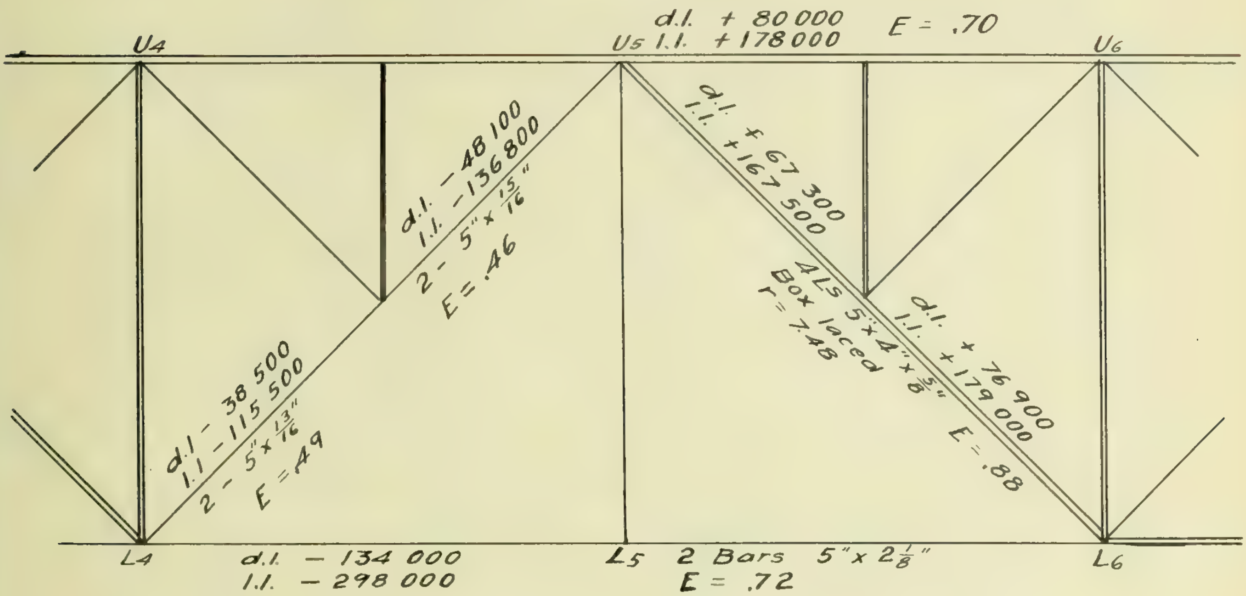
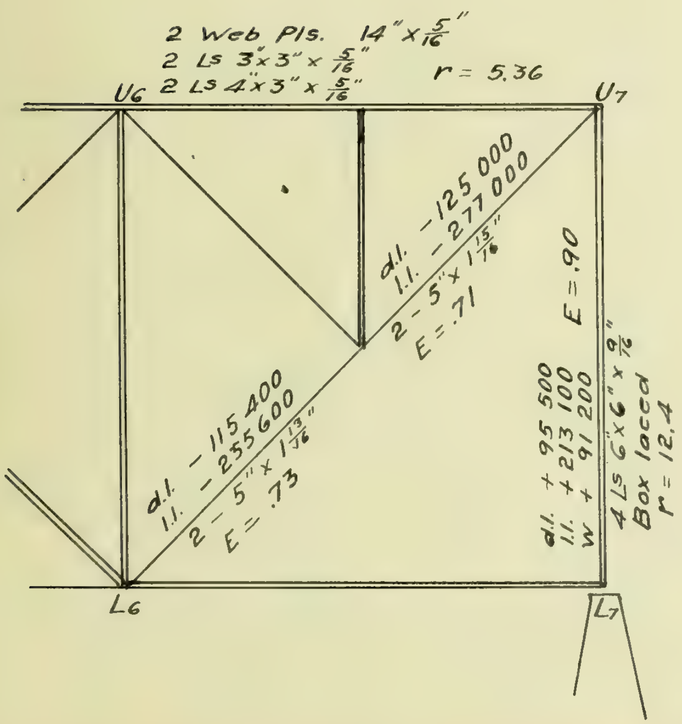
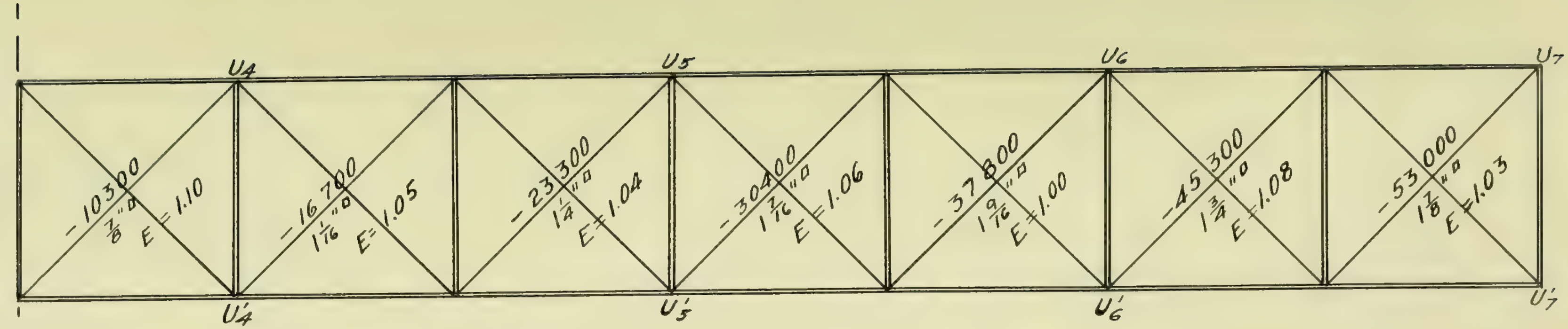


Plate No. 11

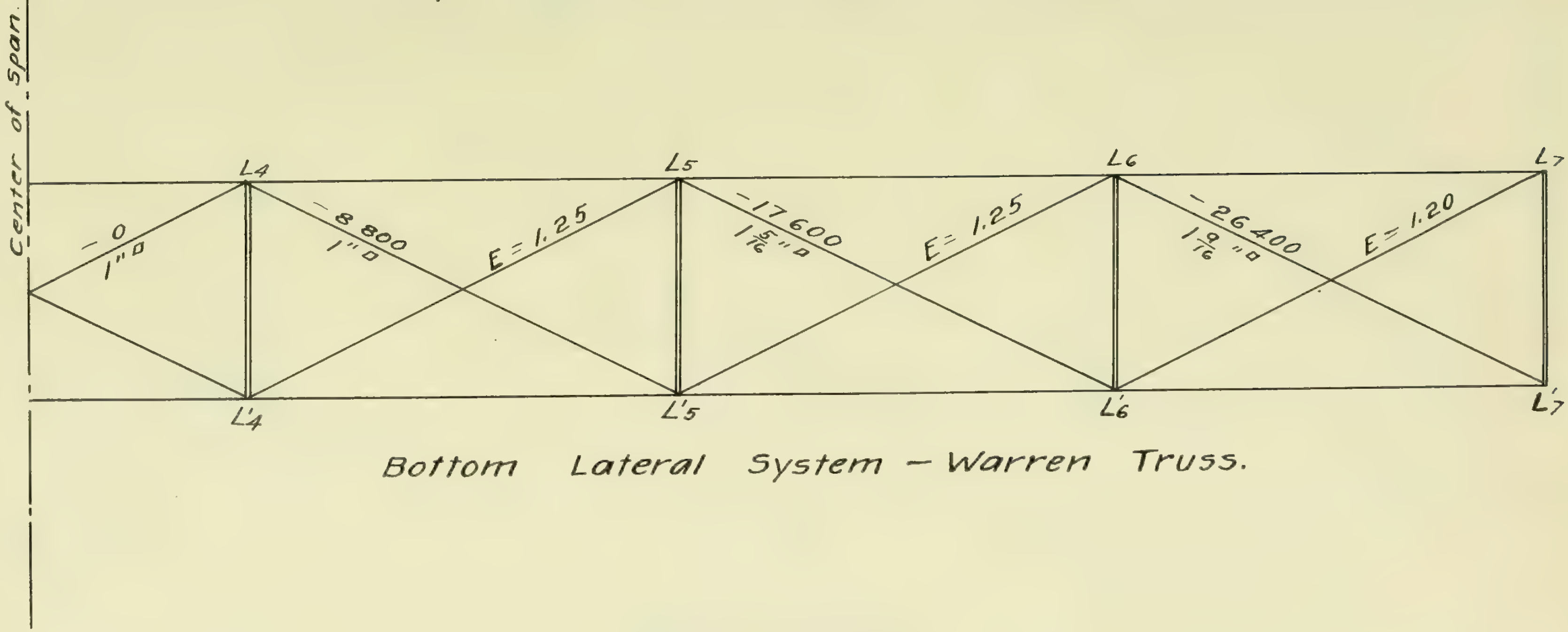
Warren Deck
(continued)







Top Lateral System – Warren Truss.

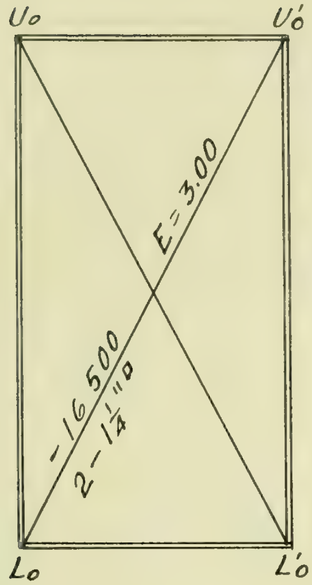


Bottom Lateral System – Warren Truss.

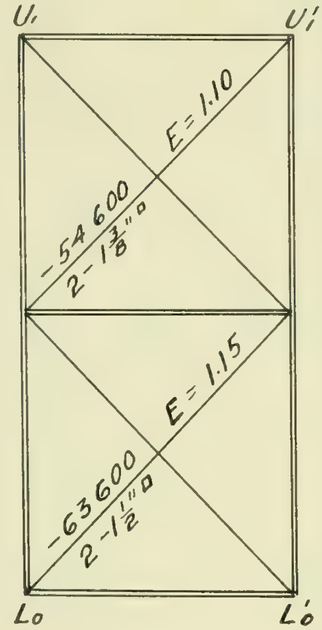
Plate No. 12

Plate No. 13

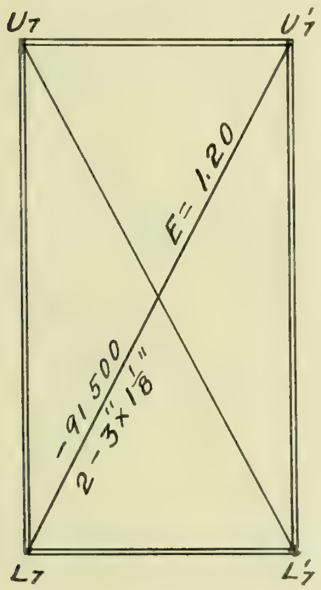
Sway Bracing – Warren Truss.



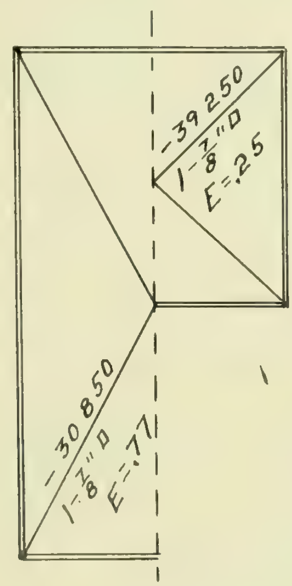
Vertical Post
North and South Piers



Batter Post
North and South Piers



Vertical Post
Channel Pier



Intermediate

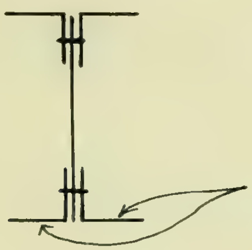
Physical Condition.

The entire bridge is in great need of a thorough scraping, followed by a coat of paint. An attempt at painting it was made in 1905, but it is evident that very little scraping was done. This is especially true of the members which are difficult of access. In a number of places on the top chord, the paint has been applied over a heavy coating of dirt and rust.

Water and dirt work through the floor of the bridge and drop upon the steel work below. The suns rays and the wind seldom reach the members directly under the floor, and so these have little effect in driving moisture and dirt from the upper members. All of the horizontal members in the upper part of the bridge are more or less corroded.

At the north end of the bridge, dirt had been permitted to pile up around the foot of bent No. 1.

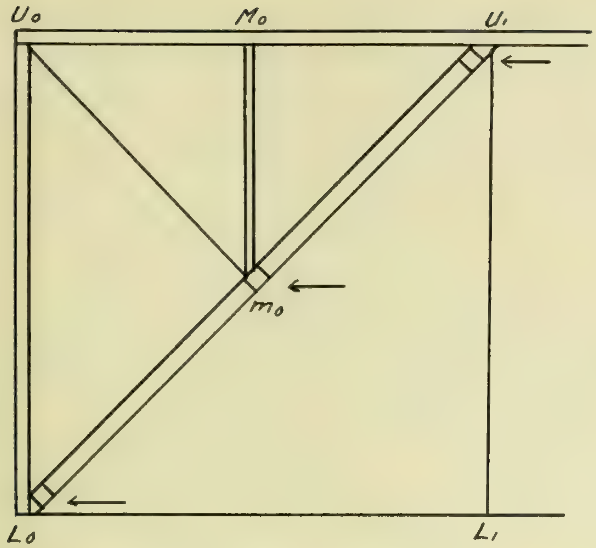
This was removed recently, and it was found that the lower angles of the lower struts had corroded until they are valueless. The angles which were originally $3'' \times 2'' \times \frac{1}{4}''$ are now only about $1'' \times 1'' \times \frac{1}{16}''$. The lower strut for this bent is composed of 4 angles laced as shown below.

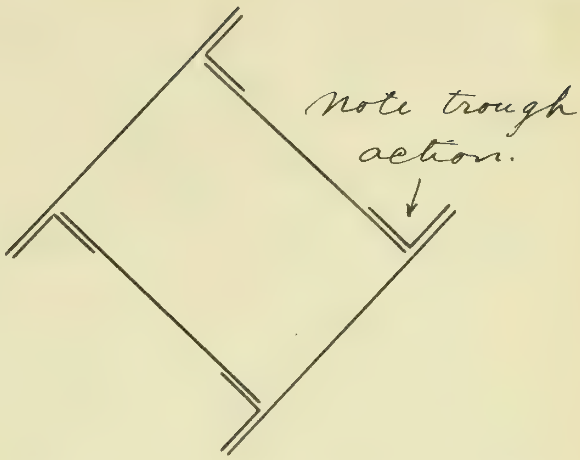


corroded until valueless.

As the posts are well anchored no serious damage has resulted.

The struts in the batter posts of the Warren trusses are badly corroded. From the diagrams, it will be noted that one angle in each of the struts has both legs turned up, and so forms a trough. The water cannot escape except by evaporation. Some of these angles have corroded until they are scarcely more than 1/16 inch thick. The angles will sooner or later have to be replaced.





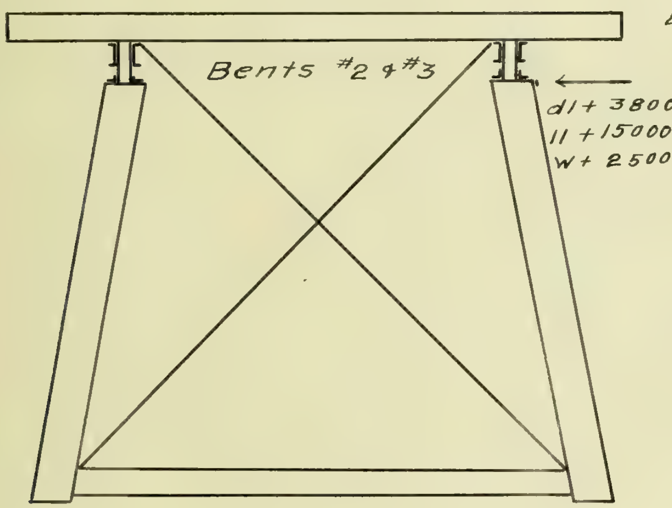
The new angles should be turned so as to be self draining.

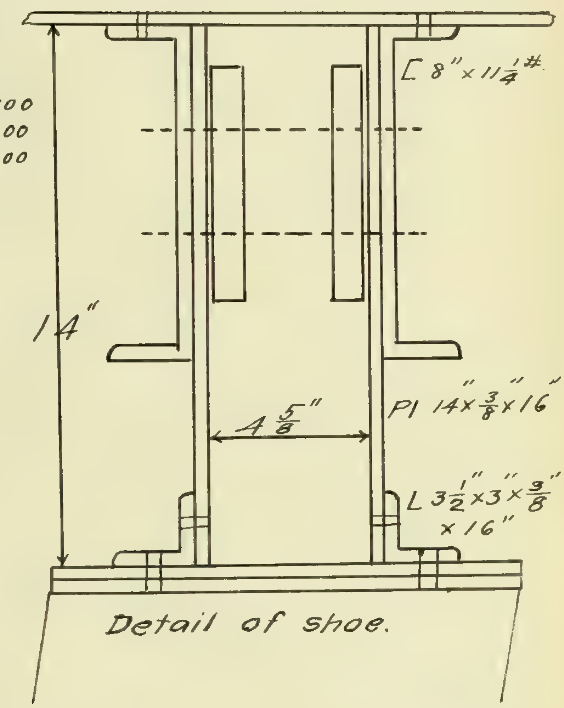
The shoes supporting the end pins of the Warren trusses are so boxed as to retain the water which is collected in them. Holes should be bored in the lower part of the box to drain off the water, or the box should be filled with cement or other material to keep out the moisture.

The rollers under the ends of the Warren trusses are of no value as they do not move. The vibrations are transferred directly to the piers. The rollers are only $2\frac{7}{8}$ inches in diameter, while they

should be $4\frac{5}{8}$ inches, according to Cooper's Specifications, paragraph 127.

Bents No 2 and No. 3, and tower No. 2 have each a very faulty design in that they have no top lateral struts. The lateral thrust due to the horizontal component of the stresses in the post, which is on a batter of 1 in 6, is taken up entirely by the webs of the top chord and the shoes supporting them (see plate below)





In the summer of 1905 it was noticed that the shoes were bent inwards, but no repairs were made. In the following December, the west shoe of bent No 3

collapsed letting the floor beam down on the end post. Last month, April 1906, the west shoe in bent No 2 collapsed. Since then the Traction Company has placed 8 inch I-Beams to take this lateral thrust.

The vibration in the long span is considerable. The amount cannot be given as there were no means at hand for determining it accurately. It seemed to the writer that at times a team trotting across the bridge, or a heavily loaded wagon bumping over the uneven floor caused a greater vibration than did the smoothly running cars.

It is not likely that the bridge is ever loaded as heavily as investigated, although it is possible that such a loading may occur. Even with this loading the bridge will not be strained beyond the elastic limit as the factor of safety is great enough to permit an efficiency of .50. With a thorough scraping and a coat of paint, with

the struts placed in the bents where needed to take up the lateral stresses, and with good rollers placed under the long spans to relieve the masonry, there need be no immediate alarm for the safety of the bridge.

It might be noted in closing that the Illinois Traction System has recently placed an order for a bridge of its own and will, in a few months, take up its tracks from the "Gilbert Street" bridge; thus relieving the loads on this structure.